FOLSOM DAM
EMBANKMENT RAISE ANALYSIS

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[b3] below elevation 427 ft.

[c] all crest elevations are equal to 480.5 ft.

2.2 Engineering Parameters

Shear strengths, permeabilities, and unit weights of embankment and foundation materials are presented in the following tables as determined from the appropriate references. Table 2.2 is a reprint of the original design strengths used for the wing dams and is presented for information and comparison purposes.

Table 2.2 Material Properties Used in Initial Design of Wing Dams.
(As originally displayed, ref. 2 Plate XV Sheet 1)

SOIL CHARACTERISTICS				
MATERIAL	IMPERVIOUS CORE	DREDGE TAILINGS	FOUNDATION	
			ZONE "A"	ZONE "B"
Dry Wt. lb/ft^3	123*	125	108	141
Moist Wt. lb/ft^3	134	133	117	150
Saturated Wt. lb/ft^3	140	144	130	151
Buoyed Wt. lb/ft^3	78	82	68	89
Tangent ϕ	0.70	0.84	0.60	1.00
Cohesion lb/ft^2	0	0	0	0
Permeability ft/day	0.5	-	10	7
*At 95% Modified A.A.S.H.O. Density				
In addition to:				
ϕ'	35	40	31	45

Notes from Definite Project Report (ref. 2):

1) No analyses were made for the sudden drawdown condition because the dredge tailings are very pervious and the impervious core is sufficiently narrow that is does not effect the stability of the section – Ref. 2, pg. 19.

2) The earthquake force of 0.05g used in the stability analyses was recommended at the Board of Consultant's Meeting held 28 to 29 September 1949 – Ref. 2, pg. 15.

Table 2.3 presents the engineering properties as determined by extensive investigations during the WES studies as given in reference 1.

Table 2.3 Current Soil Properties (ref. 1) – Right and Left Wing Dams

Material Types (Zones)	Unit Weights (lb/ft^3)		Effective Stress		Total Stress		Notes[1]
	Moist	Saturated	ϕ' (deg.)	c' (lb/ft^2)	ϕ	c	
Foundation	-	150	50	5000	-	-	
Right Wing Dam							
A – Shell	146	152	43	0	-	-	1
B - Transition	146	152	43	0	-	-	2, 7
C – Core	136	142	37	0	-	-	3
Left Wing Dam							
E – Shell	146	152	43	0	-	-	4
F – Filter	146	152	43	0	-	-	5
G - Core	136	142	37	0	-	-	6

[1]Notes from Definite Project Report (ref. 2):

1) Rock from the American River Channel – Ref. 1, Figure 4a. ...fairly dirty rockfill – Ref. 1, pg. 15. Rockfill (10 to 30% - #4) stockpiles 1, 2, 3, & 4 – Ref. 1, Table 2.

2) Unprocessed sand, gravel, and cobbles from American River channel excavation – Ref. 1, Figure 4a. Alluvial gravel dredge tailings, stockpile 7, borrow area # 7, borrow area # 8, – Ref. 1, Table 2.

3) Decomposed granite from Borrow Area No. 2 and suitable fine-grained river channel excavation – Ref. 1. Figure 4a. ...(SM) Stockpile 6, borrow area # 2 – Ref. 1, Table 2.

4) Unprocessed coarse dredged tailings from Borrow Area No. 5 – Ref. 1, Figure 6a. ...compacted gravel dredged tailings from the Blue Ravine – Ref. 1, pg. 15.

5) Minus 2" coarse dredge tailings from Borrow Area No. 5 – Ref. 1, Figure 6a.

6) Compacted decomposed granite obtained from Borrow Area No. 1 – Ref. 1, Figure 6a. ...(SM) – Ref. 1, Table 2.

7) In the design of the Right Wing Dam, the Zone A rockfill was assumed to have the same properties as the Zone B Gravel – Ref. 1, pg. 15.

Table 2.4 presents permeability values as determined in the original (1950) Definite Project Report based on testing of the borrow materials, test fill, and foundations.

Table 2.4 Permeability values from initial testing and design (ref. 2), Embankment and Foundation - Wing Dams and Dikes.

Source	Material Description	k_{min} (ft/day)	k_{max} (ft/day)	k_{avg} (ft/day)	k_{design} (ft/day)	Notes[1]
Borrow #1	Silty gravelly sand to sandy clay	.0004	.035	.01 (weighted)	-	1
Borrow #2	Silty gravelly sand to sandy clay	.00008	.32	.03 (weighted)	-	1
Borrow #4	Silty gravelly sand to clayey sand	.016	.35	.09 (weighted)	-	1
Test Fills	-	k_h = .06 k_v = .02	k_h = 2.54 k_v = 4.30	k_h = 0.48 k_v = 0.52	k_h = 2.0 k_v = 0.50	1, 2
Borrow #5	dredge tailings	370	4360	1980	note 6	1,6
Foundation	upper layer (5' thick) of topsoil and decomposed granite	k_h = .04 k_v = .24	k_h = 61.2 k_v = 11.7	k_h = 14.5 k_v = 4.2	$k_{h,v}$ = 10.0	3, 4
Foundation	underlying layer (5' thick) of dense disintegrated granite	k_h = .07 k_v = .01	k_h = 24.0 k_v = 39.6	-	$k_{h,v}$ = 7.0	3, 5

[1]Notes from Definite Project Report (ref. 2):

1) Ref. 2, pgs. 11 & 12

2) "...the results are on undisturbed material from the test fills and should be more representative of the actual conditions as obtained during construction of the embankment. Therefore, a coefficient of permeability of 0.50 feet per day (1.8×10^{-4} cm. per second) was used for design. Although there is no apparent consistent relationship between the permeability in the horizontal and vertical directions, construction procedures are conducive to stratification and a horizontal permeability four times greater than the vertical permeability was assumed." (Ref. 2, pg. 12)

3) Ref. 2, pgs. 3 & 4

4) "...no consistent relationship was indicated between the horizontal and vertical permeabilities for those samples for which the coefficient of permeability was determined in both directions. Therefore, an average coefficient of permeability of 10.0 feet per day (35×10^{-4} cm. per second) in both the horizontal and vertical directions was used as the basis of design for the topsoil and decomposed granite." (Ref. 2. pg. 4)

5) "No consistent relationship between the horizontal and vertical coefficients of permeabilities was apparent. Therefore, an average coefficient of permeability of 7.0 feet per day (25 x 10^{-4} cm. per second) based on all the tests was adopted for use in design." (Ref. 2. pg. 4)

6) "The permeability of the transition zones and the dredge tailing material is assumed infinitely higher than the impervious core." (Ref. 2, plate XX).

Shear strength and permeability properties of the Mormon Island embankment are shown in Tables 2.5 and 2.6 respectively.

Table 2.5 Mormon Island Embankment - Current Design Values

Zone	Name	Material	Effective Friction Angle ϕ'	Effective Cohesion (psf)	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)
1	Shell	minus 2" dredge tailings	43°	0	146	152
2	Transition	dredge tailings	37°	0	136	142
4	Core	Clayey sand	35°	0	125	135
A	Foundation	Tailings	41°	0	-	136
B	Foundation	Bottom	41°	0	-	147

Notes: Zone 3 (filter) was assumed to have the same strengths as Zone 4.

Table 2.6 Mormon Island Embankment - Permeabilities from Initial Testing and Design

Zone	Name	Source	Material Description	Fines %	k_v (ft/day)	k_h (ft/day)	Notes
1	Shell	Borrow Area 5	minus 2" dredge tailings	0 - 5	200	400	1
2	Transition Zone, Outer	Borrow Area 5	dredge tailings	0 - 5	25	100	2
3	Transition Zone, Inner	Borrow Area 1	Silty gravelly sand	25	0.035	0.15	3
4	Core	Borrow Area 6	Clayey sand	32	0.01	0.05	4

Notes:

1) Dredged gravels from Blue Ravine. Coefficient of permeability from reference 2.

2) Dredge tailings, minus 2-inch. Coefficient of permeability from reference 2.

3) Decomposed granite used for filter. Coefficient of permeability taken from reference 1, from tests on silty gravel to sandy clay samples, with a range of permeabilities from 0.0004 to 0.035 feet/day (Ref. 2, pgs 11 & 12) "...construction procedures are conducive to stratification and a horizontal permeability four times greater than the vertical was assumed." (Ref. 2, pg. 26.)

4) Coefficient of permeability from reference 2, Plate XXI.

3.0 Embankment and Dike Designs

Analysis was conducted to evaluate the stability of the 12' raise for the Right Wing Dam, Left Wing Dam, Mormon Island Dam, and the eight dikes. Also, analysis was conducted on the proposed Mooney Ridge dike. Raise designs were based on maintaining seismic deformations for the new sections equal to or less than the values as reported in the WES reports (references 1, 3, 4, 5) for the existing sections. The Makdisi-Seed seismic deformation technique was used for the analysis.

3.1 Right Wing Dam

The right wing dam was analyzed for slope stability at approximate station 260+00 and a plan view of the right wing dam is shown in Figure 3.1. It is important to note that the design cross sections are moderately different than the as-built sections with the as-built drawing shown Figure 3.2.

Seepage analyses were conducted to determine the piezometric line using the computer program GMS/SEEP2D. The gross pool elevation (466 feet) does not change with the embankment raises. Changes are being made only to the flood control pool elevations with the existing being at 470 feet, and new flood control pools at elevations 474, 478, 482, and 487 feet.

Permeability values of the embankment materials were determined from the Definite Project Report and are shown in Table 3.1. The shell and transition materials were designated as free-draining in the report. The same permeability values were assumed for the new materials.

Table 3.1 Permeability Values Used for Analysis - Existing Dam

Zone	Name	Permeability (ft/day)	
		Horizontal	Vertical
A	Shell	400	200
B	Transition	400	200
C	Core	2	0.5

Steady state seepage analyses were conducted for various pool elevations. Expected durations of the flood control pool elevations will most likely not produce steady state seepage conditions, but such assumptions were made for conservatism. Seepage analyses for the existing gross and flood control pools are shown in Figures 3.3 and 3.4. It should be particularly noticed that the piezometric line does not substantially penetrate the downstream transition or shell zones. The seepage analyses of the new flood control pool elevations are shown in Figure 3.5 through 3.8 and it can be seen that the piezometric surface does not significantly penetrate the downstream transition zone, and thus does not adversely effect the stability of the downstream section.

Seismic stability analyses were conducted to determine the design slopes for the 12-foot raise. The 12-foot raise consists of a 3.5-foot concrete parapet (flood) wall with 8.5 feet of earth fill (Figure 3.9). The computer program UTEXAS4 was used for the analysis. A crest width of 25 feet will be used for the raised section (EM 1110-2-2300, para. 4-2, "Depending upon the height of the dam the minimum top width should be between 25 and 40 feet"), the existing crest width is equal to 30 feet. As recommended in reference 1, conservatively for seismic analyses, the core strengths (tan ϕ') were reduced by 20 percent for materials below the piezometric line. The current stability model was checked against the yield acceleration results from reference 1 and excellent comparative results were obtained.

Deformations were computed using the Makdisi-Seed method to evaluate the impacts of the raised sections. Embankment periods, peak ground accelerations, and crest accelerations, etc., were assumed not to change dramatically for the 12-foot raise and those given in reference 1 were used for the current analysis. To compute and compare deformations, slip surfaces were chosen such that they are tangent to elevation 456.5 feet (1/5 of existing height) and pass through the opposite crest hinge point.

The results of yield acceleration analysis are shown in Figures 3.10 through 3.14. The existing sections were analyzed for both upstream and downstream failure surfaces as shown in Figures 3.10 and 3.11. The raised sections were also analyzed for both upstream and downstream failure surfaces as shown in Figures 3.12 and 3.13. A plot of the determined seismic coefficients for all 4 cases is shown in Figure 3.14 and detailed deformation calculations are shown in Figure 3.15. Briefly, the resulting seismic deformations are shown in Table 3.2 and it can be seen that all deformations are less than 1-inch and considered acceptable for the maximum credible earthquake.

Table 3.2 Seismic Slope Stability Results – Right Wing

Case	Height	Yield Acc. (g)	Seismic Deformations (in)
Existing - Upstream	1/5	0.387	0.37
Existing - Downstream	1/5	0.465	0
12' Raise – Upstream	1/5	0.328	0.78
12' Raise – Downstream	1/5	0.371	0.39

Examination of seismic deformations and yield accelerations analyses at other heights (2/5 through 5/5) demonstrate, as anticipated, that the effects of the 12 foot raise are not detrimental or even felt at these larger depths.

3.2 Left Wing Dam

The left wing dam has slightly different zoning characteristics than the right wing dam. However, when the material properties are taken into account, the cross sections between the right and left wings are essentially the same. Therefore, the design prepared for the right wing dam is applicable to the left wing dam. Existing plan view and cross sections of the left wing dam are shown in Figures 3.16 and 3.17 respectively.

3.3 Mormon Island Auxiliary Dam

This feature of the project was analyzed at station 446+00 for the 12' raise. Current material properties are shown in Table 3.3 as determined from the WES reports. Conservatively and as recommended in the WES report, the shear strength (tan ϕ') of the core material (zone 4) was reduced by 20 percent due to earthquake shaking and the generation of excess pore pressure.

Table 3.3 Current Design Values

Zone [a]	Name	Effective Friction Angle ϕ'	Effective Cohesion (psf)	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)
1	Shell	43°	0	146	152
2	Transition	37°	0	136	142
4	Core	35/29° [b]	0	125	135

[a] Zones 3 and 4 were assumed to have the same properties.
[b] Strength (tan ϕ') reduced by 20 percent for materials below the piezometric line.

The piezometric line was determined by seepage analysis as shown in Figure 3.18. Seismic deformation analysis of the existing structure resulted in deformations of the upstream upper 1/5 failure surface (Figure 3.19) of approximately 3 inches and less than 1 inch for the downstream upper 1/5 failure surface. The deformations correlated well with the results of the WES reports. Due to the higher crest acceleration and period of the embankment, when compared to the right and left wings, larger deformations will occur. The design slopes where therefore chosen to be slightly flatter than those used for the wing dams. The proposed design upstream slopes for the 12' raise (8.5' earthfill, 3.5' parapet wall) are equal to 2H:1V and the proposed downstream slopes are equal to 1.75H:1V (Figure 3.20). The resulting seismic deformations for the raise are less than 3 inches for the upstream upper 1/5 failure surface (Figure 3.21) and less than 1inch for the downstream upper 1/5 failure surface. These deformations for the maximum credible earthquake are considered acceptable.

3.4 Dikes

Dikes were designed using the embankment and seismic properties for Dike 5 as determined in reference 5. Dike as-built cross-sections are shown in Figure 3.22. Designs were based on maintaining relatively identical seismic deformations for the raised section as compared to the existing section. Sections were evaluated for both tall dikes, as in Dike 5, and shorter dikes, as in Dike 1. The piezometric line was determined by seepage analysis as shown in Figure 3.23. Most of the dikes are essentially homogeneous except for dikes 5 & 7 where the "core material" was compacted to a higher percentage than the surrounding material.

Table 3.4 Dike 5 - Current Design Values

Zone[a]	Name	Effective Friction Angle ϕ'	Effective Cohesion (psf)	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	k_v (ft/day)	k_h (ft/day)
1	Shell	38/32°[b]	0	146	152	0.5	2
2	Core	38/32°[b]	0	125	135	0.1	0.5

[a] Materials from the same source but core is compacted at higher percentages.
[b] Strength (tan ϕ') reduced by 20 percent for materials below the piezometric line.

Detailed seismic deformation calculations are shown in Figure 3.15. The design slopes for the raised section were determined to be 2.25H:1V for the upstream slope and 1.75H:1V for the downstream slope as shown in Figures 3.24 and 3.25 respectively, for the tall and short dikes. The deformations for the existing upstream failure surface (Figure 3.26) are equal to 14.33 inches as compared to 14.84 inches for the upstream failure surface (Figure 3.27) for the raised section. The existing downstream deformation is equal to 10.57 inches as compared to 10.72 inches for the raised section. These deformations for the maximum credible earthquake are considered acceptable.

4.0 Conclusions

Designs were prepared for the 12' raise proposed for Folsom Dam and appurtenances as part of the American River Watershed - Long Term Study. It is important to note that, due to project schedule constraints and lengthy environmental clearances, borrow site analyses have not been completed and designs are based on current properties of the existing embankments and dikes. In fact, final borrow site determination will not occur until the project is authorized for PED (pre-construction, engineering, and design). The proposed slopes must be verified once borrow site properties are determined. Design slopes were determined by using existing soil properties, and by comparing seismic deformations of the raised versus the existing sections and maintaining nearly identical deformation magnitudes. The proposed slopes for the 12' raised sections are shown in Table 4.1. The slopes will also apply to embankment raises of lesser height. The 12' raise consists of 8.5' of earthfill and 3.5' of parapet concrete wall -- except for Mooney Ridge dike which is composed entirely of earthfill. The Mooney Ridge dike was designed conservatively using engineering judgment since environmental and interagency clearances for foundation explorations are still pending.

Table 4.1 Proposed slopes for the 12' raise

Feature	Crest Width (feet)	Upstream Slope __H:1V	Downstream Slope __H:1V
Left Wing Embankment	25	1.75	1.5
Right Wing Embankment	25	1.75	1.5
Mormon Island Auxiliary Embankment	25	2.0	1.75
Dikes 1 - 8	20	2.25	1.75
Mooney Ridge Dike	20	2.5	2.25

Deformations for the maximum credible earthquake are considered acceptable for all features.

References:

1) U. S. Army Corps of Engineers, Waterways Experiment Station, “Seismic Stability Evaluation of Folsom Dam and Reservoir Project, Report 6, Right and Left Wing Dams,” April 1989, Geotechnical Laboratory, Technical Report GL-87-14.

2) U. S. Army Corps of Engineers, Sacramento District, “Definite Project Report - Soils Data and Embankment Design,” Folsom Project, American River California, Part IV Dam and Appurtenances, Appendix C, Section I, 20 February 1950.

3) U. S. Army Corps of Engineers, Waterways Experiment Station, “Seismic Stability Evaluation of Folsom Dam and Reservoir Project, Report 4, Mormon Island Auxiliary Dam - Phase I,” December 1990, Geotechnical Laboratory, Technical Report GL-87-14.

4) U. S. Army Corps of Engineers, Waterways Experiment Station, “Seismic Stability Evaluation of Folsom Dam and Reservoir Project, Report 8, Mormon Island Auxiliary Dam - Phase II,” August 1992, Geotechnical Laboratory, Technical Report GL-87-14.

5) U. S. Army Corps of Engineers, Waterways Experiment Station, “Seismic Stability Evaluation of Folsom Dam and Reservoir Project, Report 5, Seismic Stability Evaluation of Dike 5,” November 1988, Geotechnical Laboratory, Technical Report GL-87-1